

## **Chapter 5**

### **Subsurface Investigations**

#### **5-1. Background**

Subsurface investigations require use of equipment to gain information below the ground surface. The equipment is typically invasive and requires disturbance of the ground to varying degrees. Most of these exploration techniques are relatively expensive and therefore should be carefully planned and controlled to yield the maximum amount of information possible. It should be kept in mind that the quality of the information produced can vary significantly. If procedures are not followed carefully and data not interpreted properly, radically different conclusions can be reached. For example, poor drilling techniques could produce samples that might yield lower strength values. Therefore, only competent, senior geotechnical personnel should be charged with planning a subsurface investigation, and only qualified geotechnical professionals and technicians should do the drilling and data collecting, reducing, analyzing, and interpreting.

#### **5-2. Location of Investigations**

An important piece of information for all geotechnical investigations that seems obvious but commonly not given sufficient attention is the accurate determination of the location of investigation. It is always preferable to select boring and test pit locations that fully characterize geotechnical conditions. Although correlation of information from offsite may be technically defensible, because of variability of geologic materials, the legal defensibility of a piece of information is commonly lost if it is even slightly removed from the site. Of course, it is not always possible to locate a boring on a structure because of obstacles or right-of-entry difficulties. Heavily urbanized areas present particular difficulties in these aspects. However, it is important to keep in mind that correlations and interpretations may be subject to later scrutiny should a change of conditions claim be filed. All locations should be determined using either conventional surveying methods or by a GPS (EM 1110-1-1003). A GPS has the significant advantage of having the positional information downloaded directly into a GIS.

#### **5-3. Protection of the Environment**

*a.* After the locations for field investigations work have been determined, routes of access to the area and the specific sites for borings and excavations should be selected with care to minimize damage to the environment. Environmental engineering aspects of civil works projects are discussed in EM 1110-2-1202, -1204, -1205, and -1206 and Keller (1992). Operation of equipment will be controlled at all times and the extent of damaged areas will be held to the minimum consistent with the requirements for obtaining adequate data. Local laws pertaining to permissible levels of sediment flow from the site should be investigated. After the exploratory sites have served their purpose, the disturbed areas will be restored to a natural appearance. All borings and test pits should be backfilled in accordance with state environmental regulations.

*b.* Most states are now the primary regulatory agency for ground water quality assurance. As part of this responsibility many now require the certification of drillers. These regulations primarily apply to water well installation, but they may also apply to investigation programs. Ground water quality assurance has been the subject of considerable discussion from the standpoint of Federal Government responsibility for compliance with these regulations. Generally, Government drillers are not required to

have state certification but, in some instances, may be forced to comply for political reasons. This is not a clear-cut issue, and it should be resolved before beginning a drilling program.

c. The Federal Government has responsibility to ensure that environmental consciousness is maintained during the conduct of geotechnical investigations. Unfortunately, drilling rigs are inherently dirty. Proper maintenance of drilling rigs will minimize this problem. For HTRW exploratory drilling, drilling rigs must be steam cleaned and all tools, equipment, and personnel decontaminated in accordance with procedures established in the quality assurance and control (QAAC) plan. Fluids used in drilling operations, be they hydrocarbons that have leaked from the hydraulic system or a constituent of a drilling mud, are potentially toxic and should be controlled or eliminated wherever possible. EM 1110-1-4000 discusses requirements for maintenance and operation of drilling equipment at USACE HTRW sites. Aller et al. (1989) provide further guidance on acceptable design and installation of monitoring wells.

## *Section I*

### *Borings*

#### **5-4. Major Uses**

Borings are required to characterize the basic geologic materials at a project. The major uses for which borings are made are as follows:

- a. Define geologic stratigraphy and structure.
- b. Obtain samples for index testing.
- c. Obtain ground water data.
- d. Perform in situ tests.
- e. Obtain samples to determine engineering properties.
- f. Install instrumentation.
- g. Establish foundation elevations for structures.
- h. Determine the engineering characteristics of existing structures.

Borings are classified broadly as disturbed, undisturbed, and core. Borings are frequently used for more than one purpose, and it is not uncommon to use a boring for purposes not contemplated when it was made. Thus, it is important to have a complete log of every boring, even if there may not be an immediate use for some of the information. If there is doubt regarding the range of borehole use or insufficient information to determine optimum borehole size, then the hole should be drilled larger than currently thought needed. A slightly larger than needed borehole is considerably less expensive than a second borehole.

#### **5-5. Boring and Sampling Methods**

a. *Common methods discussed.* Many methods are used to make borings and retrieve samples. Some of the more common methods are discussed in the following paragraphs. Many of these are also

discussed in detail in Chapter 3, Appendix F; Das (1994); Hunt (1984); and Aller et al. (1989). Some factors that affect the choice of methods are:

- (1) Purpose and information required.
- (2) Equipment availability.
- (3) Depth of hole.
- (4) Experience and training of available personnel.
- (5) Types of materials anticipated.
- (6) Terrain and accessibility.
- (7) Cost.
- (8) Environmental impacts.
- (9) Disruption of existing structure.

*b. Auger borings.* Auger borings provide disturbed samples that are suitable for determining soil type, Atterberg limits, Proctor testing, and other index properties but generally give limited information on subsoil stratification, consistency, or sensitivity. Auger borings are most useful for preliminary investigations of soil type, advancing holes for other sampling methods, determining depth to top of rock, and for monitor well installation in soils. Auger borings can be made using hand, helical, barrel, hollow-stem, or bucket augers. Auger samples are difficult to obtain below the ground water table, except in clays. However, hollow-stem augers with a continuous split barrel sampler can retrieve some unconsolidated material from below the water table. Paragraph 3-4, Appendix F, describes the types of augers used in subsurface exploration. Paragraph 8-2, Appendix F, discusses sampling procedures when augering.

(1) Truck-mounted auger rigs currently come equipped with high yield and high tensile strength steel augers. New hydraulics technology can now apply torque pressures upward of 27,000 Nm (20,000 ft lb). With this amount of torque, augers are capable of boring large size holes and of being used in soft rock foundation investigations. Because augers use no drilling fluids, they are advantageous for avoiding environmental impacts. Appendix F, paragraph 3-3, describes auger drilling rigs. Another advantage of using augers is the ability (using hollow stems) for soil sampling, i.e., taking undisturbed samples below the bit.

(2) Currently, many drilling rigs are actually a combination of auger/core/downhole hammer units. A hollow-stem auger has the “drill through” capability (i.e., the auger can drill to refusal, then a wireline core barrel and drill rods can be inserted to finish the hole). The auger acts as a temporary casing to prevent caving of the softer materials as sampling progresses. However, the augers are not water tight and water loss should be anticipated. Hollow-stem augers should not be used as temporary casing in areas where HTRW is anticipated. Temporary steel casing driven into the surface of competent bedrock or PVC casing permanently grouted into the competent bedrock surface is required when HTRW is anticipated.

c. *Drive borings.* Drive borings provide disturbed samples that contain all soil constituents, generally retain natural stratification, and can supply data on penetration resistance. Drive boring is a nonrotating method for making a hole by continuous sampling using a heavy wall drive barrel. Push, or drive, samplers are of two types: open samplers and piston samplers. Open samplers have a vented sampler head attached to an open tube that admits soil as soon as the tube is brought in contact with the soil. Some open samplers are equipped with a cutting shoe and a sample retainer. Piston samplers have a movable piston located within the sampler tube. The piston helps to keep drilling fluid and soil cuttings out of the tube as the sampler advances. The piston also helps to retain the sample in the sampler tube. Where larger samples are required, the most suitable drill for this method is the cable tool rig. The cable tool rig has the capability to provide a downward driving force (drill stem on drive clamps) to make a hole and an upward force (drilling jars) to remove the drive barrel from the hole.

(1) Vibratory samplers offer a means of obtaining disturbed samples of saturated, cohesionless soils rapidly and with relatively inexpensive equipment (Appendix F). The simplest devices consist of a small gasoline engine providing hydraulic power to a vibrating head clamped to aluminum tubing secured on a tripod. The rapid vibrations within the head drives the sampling tube into the ground and forces the soil up into the tube. A rubber packer secured into the open end of the sampling tube after driving creates a seal to retain the sample as the tube is withdrawn with a hand winch.

(2) Another device, the Becker hammer drill, was devised specifically for use in sand, gravel, and boulders by Becker Drilling, LTD, Canada. The Becker drill uses a diesel-powered pile hammer to drive a special double-wall, toothed casing into the ground. Drilling fluid is pumped through an annulus to the bottom of the hole where it forces cuttings to the surface through the center of the casing. The cuttings are collected for examination. Becker drill casings are available in 14-cm (5.5-in.), 17-cm (6.6-in.), and 23-cm (9.0-in.) outside diameters (OD), with sampling inside diameters (ID) of 8.4 cm (3.3 in.), 10.9 cm (4.3 in.), and 15.2 cm (6.0 in.), respectively. Paragraph 5-23 and Appendix H describe Becker penetration test procedures. Appendix F, paragraph 3-3, discusses the Becker hammer drilling equipment and operation.

(3) The Standard Penetration Test (SPT) method of drive boring, described in ASTM D 1586-84 (ASTM 1996b), is probably the most commonly used method for advancing a hole by the drive method. Slight variations of this method, primarily concerning the sampling interval, cleanout method, and the refusal criteria exist from office to office but the fundamental procedure follows the ASTM standard. Appendix G presents procedures for SPT sampling and testing. Appendix G is compatible with the ASTM D 1586-84 standard and provides additional guidance in evaluating the test data. In this method, a standard configuration, 5-cm (2-in.) OD split barrel sampler at the end of a solid string of drill rods is advanced for a 0.45-m (1.5-ft) interval using a 623-Newton (N) (140-lb) hammer dropped through a 76-cm (30-in.) free fall. The blows required to advance the hole for each 15-cm (6-in.) interval are recorded on ENG Form 1836. The standard penetration resistance, or “N” value, is the sum of the blows required for the second and third 15-cm (6-in.) drives. The hole is then cleaned or reamed to the top of the next interval to be sampled and the procedure is repeated. Refusal is generally defined as 50 blows per 15 cm (per half foot) of penetration. When used to define the top of rock, great care and close examination of samples are required to minimize uncertainties. A few of the applications of SPT data are listed in paragraph 5-23a. This impact method may also be used with larger sample tubes and heavier hammers. Correlation studies to normalize data from larger holes to the SPT have been performed but are not completely reliable. The Becker hammer drill data can provide correlations of soil density and strength in coarse-grained soils similarly to the SPT test in finer-grained soils (paragraph 5-23a).

(4) Drive borings can be advanced quickly and economically with hollow-stem augers using a “plug” assembly that is either manually or mechanically set in the opening at the end of the auger string and then removed prior to sampling. Removal is commonly facilitated using a wire line system of retrieval. Where overburden prohibits the use of augers to advance the boring due to boulders or resistant rock lenses or ledges, other methods can be used. Traditionally, a roller rock bit using drilling mud will advance the hole at a modest cost in time and dollars. Where extremely difficult drilling conditions exist, an ODEX (eccentric reamer) down-the-hole air hammer system or other coring advancer apparatus can be used to penetrate the toughest boulders or ledges while still permitting the use of standard penetration or even undisturbed sampling to be conducted.

*d. Cone penetration borings.* The Cone Penetration Test (CPT) or Dutch cone boring is an in situ testing method for evaluating detailed soil stratigraphy as well as estimating geotechnical engineering properties (Schmertmann 1978a). The CPT involves hydraulically pushing a 3.6-cm (1.4-in.) diam special probe into the earth while performing two measurements, cone resistance and sleeve friction resistance. The probe is normally pushed from a special heavy duty truck but can also be performed from a trailer or drilling rig. Because of the weight of the truck or trailer needed to conduct CPT borings, access to soft ground sites is limited. Recent developments in CPT technology make it possible to retrieve physical soil samples and ground water or soil-gas samples with the same drive string used to perform the cone penetration test. CPT vehicles with push capacities up to 267 kiloNewtons (kN) (30 tons) have been developed. The Tri-Service Site Characterization and Analysis Penetrometer System (SCAPS), which is used to detect underground HTRW, is a technical variation of the CPT. The use of SCAPS reduces the time and cost of site characterization and restoration monitoring by providing rapid onsite real-time data acquisition/processing (i.e., in situ analysis) and onsite 3-D visualization of subsurface stratigraphy and regions of potential contamination. The Triservices operate several SCAPS vehicles including those of the U.S. Army Engineer District, (USAED) Kansas City, Savannah, and Tulsa, and the U.S. Army Engineer Waterways Experiment Station (USAEWES). Additional discussion of CPT is given in paragraph 5-23f.

*e. Undisturbed borings.* Appendix F, Chapters 5 and 6, discuss procedures for undisturbed sampling of soils. True “undisturbed” samples cannot be obtained because of the adverse effects resulting from sampling, shipping, or handling. However, modern samplers, used with great care, can obtain samples that are satisfactory for shear strength, consolidation, permeability, and density tests provided the possible effects of sample disturbance are considered. Undisturbed samples can be sliced to permit detailed study of subsoil stratification, joints, fissures, failure planes, and other details. Undisturbed samples of clays and silts can be obtained as well as nearly undisturbed samples of some sands.

(1) There are no standard or generally accepted methods for undisturbed sampling of noncohesive soils. One method that has been used is to obtain 7.6-cm (3-in.) Shelby (thin-wall) tube samples, drain them, and then freeze them prior to transporting them to the laboratory. Another method used consists of in situ freezing, followed by sampling with a rotary core barrel. Care is necessary in transporting any undisturbed sample, and special precautions must be taken if transporting sands and silts. For both methods, disturbance by cryogenic effects must be taken into account. Fixed-piston (Hvorslev) samplers, wherein a piston within a thin-walled tube is allowed to move up into the tube as the sampler is pushed into the soil, are adapted to sampling cohesionless and wet soils (Appendix F, paragraph 5-1a(2)).

(2) Undisturbed borings are normally made using one of two general methods: push samplers or rotary samplers. Push sampling types involve pushing a thin-walled tube using the hydraulic system of the drilling rig, then enlarging the diameter of the sampled interval by some “cleanout” method before beginning to sample again. Commonly used systems for push samples include the drill-rig drive,

whereby pressure is applied to a thin-walled (Shelby) sampling tube through the drill rods, the Hvorslev fixed-piston sampler, and the Osterberg hydraulic piston sampler. Rotary samplers involve a double tube arrangement similar to a rock coring operation except that the inner barrel shoe is adjustable but generally extends beyond the front of the rotating outer bit. This minimizes the disturbance to the sample from the drilling fluid and bit rotation. Commonly used rotational samplers include the Denison barrel and the Pitcher sampler. The Pitcher sampler has an inner barrel affixed to a spring-loaded inner sampler head that extends or retracts relative to the cutting bit with changes in soil stiffness. Drilling fluids are commonly used with rotary drilling equipment to transport cuttings to the surface and to increase the stability of the borehole. Chapter 4 of Appendix F discusses the types, preparation, and use of drilling fluids. The standard for thin-walled tube sampling of soils is ASTM D 1587-94 (ASTM 1996c), "Standard Practice for Thin-Walled Tube Sampling of Soils."

*f. Rock core boring.* Cored rock samples are retrieved by rotary drilling with hollow core barrels equipped with diamond- or carbide-embedded bits. The core is commonly retrieved in 1.5- to 3-m (5- to 10-ft) lengths. The "N" size hole (approximately 75 mm or 3 in.) is probably the core size most widely used by the Corps of Engineers for geotechnical investigations and produces a satisfactory sample for preliminary exploration work and, in many instances, for more advanced design studies. Other hole sizes, including B (approx 60 mm or 2.3 in.) and H (approximately 99 mm or 4 in.), are also quite satisfactory for geotechnical investigations. The decision on hole size should be based upon anticipated foundation conditions, laboratory testing requirements, and the engineering information desired. A double- or triple-tube core barrel is recommended because of its ability to recover soft or broken and fractured zones. The use of wireline drilling, whereby the core barrel is retrieved through the drill rod string, eliminates the need to remove the drill rods for sampling and saves a great deal of time in deep borings. Table 5-1 summarizes core and hole sizes commonly used in geotechnical studies. The rock boring is advanced without sampling using solid bits, including fishtail, or drag, bits, tri-cone and roller rock bits, or diamond plug bits.

(1) Most rock boring in the Corps of Engineers is accomplished using truck-mounted rotary drilling rigs. Skid-mounted rigs are also sometimes used in areas with poor access. Rotary drilling rigs are driven by the power takeoff from the truck engine or by independent engines. Boreholes are advanced by rotary action coupled with downward pressure applied to the drill bit and the cleaning action of the drilling fluid. Two types of pulldown mechanisms are normally used. Truck-mounted rotary drilling rigs equipped with a chain pulldown drive mechanism are capable of drilling to depths of 60 to 300 m (200 to 1,000 ft). Hydraulic feed drive rotary drilling rigs are capable of drilling to depths of 150 to 750 m (500 to 2,500 ft).

(2) Core recovery in zones of weak or intensely fractured rock is particularly important because these zones are typically the critical areas from the standpoint of foundation loading and stability. The use of larger-diameter core barrels in soft, weak, or fractured strata can improve core recovery and provides a statistically better size sample for laboratory testing. The advantages of larger cores must be weighed against their higher costs.

(3) Although the majority of rock core borings are drilled vertically, inclined, and horizontally oriented, borings may be required to adequately define stratification, jointing, and other discontinuities. A bias exists in the data favoring discontinuities lying nearly perpendicular to the boring. Discontinuities more nearly parallel to the boring are not intersected as often, and therefore, their frequency will appear to be much lower than it actually is. Inclined borings should be used to investigate steeply inclined jointing in abutments and valley sections for dams, along spillway and tunnel

**Table 5-1**  
**Typical Diamond Core Drill Bit and Reaming Shell Dimensions**

Size	Bit Size		Reaming Shell
	OD, mm (in.)	ID, mm (in.)	OD and hole diam, mm (in.)
<u>"W" Group - "G" and "M" Design</u>			
EWG (EWX), EWM	37.3 (1.470)	21.5 (0.845)	37.7 (1.485)
AWG (AWX), AWM	47.6 (1.875)	30.1 (1.185)	48.0 (1.890)
BWG (BWV), BWM	59.6 (2.345)	42.0 (1.655)	59.9 (2.360)
NWG (NWV), NWM	75.3 (2.965)	54.7 (2.155)	75.7 (2.980)
HWG	98.8 (3.890)	76.2 (3.000)	99.2 (3.907)
<u>"W" Group - "T" Design</u>			
RWT	29.5 (1.160)	18.7 (0.735)	29.9 (1.175)
EWT	37.3 (1.470)	23.0 (0.905)	37.7 (1.485)
AWT	47.6 (1.875)	32.5 (1.281)	48.0 (1.890)
BWT	59.6 (2.345)	44.4 (1.750)	59.9 (2.360)
NWT	75.3 (2.965)	58.8 (2.313)	75.7 (2.980)
HWT	98.8 (3.890)	81.0 (3.187)	99.2 (3.907)
<u>Large-Diameter Design</u>			
2-3/4 X 3-7/8	97.5 (3.840)	68.3 (2.690)	98.4 (3.875)
4 X 5-1/2	138.1 (5.435)	100.8 (3.970)	139.6 (5.495)
6 X 7-3/4	194.4 (7.655)	151.6 (5.970)	196.8 (7.750)
<u>Wireline Sizes</u>			
AQ		27.0 (1 <sup>1</sup> / <sub>16</sub> )	48.0 (1 <sup>57</sup> / <sub>64</sub> )
BQ		36.5 (1 <sup>7</sup> / <sub>16</sub> )	60.0 (2 <sup>23</sup> / <sub>64</sub> )
NQ		47.6 (1 <sup>7</sup> / <sub>8</sub> )	75.8 (2 <sup>63</sup> / <sub>64</sub> )
HQ		63.5 (2 <sup>1</sup> / <sub>2</sub> )	96.0 (3 <sup>25</sup> / <sub>32</sub> )
PQ		85.0 (3 <sup>11</sup> / <sub>32</sub> )	122.6 (4 <sup>53</sup> / <sub>64</sub> )

alignment, and in foundations for other structures. In nearly vertical bedding, inclined borings can be used to reduce the total number of borings needed to obtain core samples of all strata.

(4) If precise geological structure is to be evaluated from core samples, techniques involving oriented cores are required. In these procedures, the core is scribed or engraved with a special drilling tool (Goodman 1976) so that its orientation is preserved. In this manner, both the dip and strike of any joint, bedding plane, or other planar surface can be ascertained. A more common procedure for obtaining dip and strike of structural features is the use of borehole photography or television. If the orientation of bedding is consistent across the site, it can be used to orient cores from borings which are angled to this bedding. Once oriented, the attitudes of discontinuities can be measured directly from the core.

(5) Large-diameter borings or calyx holes, 0.6 m (2 ft) or more in diameter, are occasionally used in large or critical structures. Their use permits direct examination of the sidewalls of the boring or shaft and provides access for obtaining high quality undisturbed samples. Direct inspection of the sidewalls may reveal details, such as thin, weak layers or shear planes that may not be detected by continuous undisturbed sampling. Large-diameter borings are produced with augers in soil and soft rock, and with large-diameter core barrels in hard rock.

## 5-6. Drilling in Embankments

The Corps of Engineers developed a special regulation concerning drilling operations in dam and levee embankments and their soil foundations (ER 1110-1-1807). In the past, compressed air and other drilling fluids have been used as circulating media to remove drill cuttings, stabilize bore holes, and cool and lubricate drilling bits. There have been several incidents of damage to embankments and foundations when drilling with air, foam, or water as the circulating medium. Damage has included pneumatic fracturing of the embankment while using air or air with foam, and erosion of embankment or foundation materials and hydraulic fracturing while using water. The new ER establishes a policy for drilling in earth embankments and foundations and replaces ER 1110-1-1807. The following points summarize the guidance provided in the new document:

- a. Personnel involved in drilling in dam and levee embankments shall be senior and well qualified. Designs shall be prepared and approved by geotechnical engineers or engineering geologists. Drillers and “mud” specialists shall be experts in their fields.
- b. Drilling in embankments or their foundations using compressed air or other gas or water as the circulating medium is prohibited.
- c. Cable tool, auger, and rotary tool are recommended methods for drilling in embankments. One Corps District reports using a churn drill (a cable tool rig) to sample the clay core of a dam to a depth of 90 m (300 ft) with no damage to the core. If the cable tool method is used, drilling tools must be restricted to hollow sampling (drive) barrels in earth embankment and overburden materials. Appendix F, page 3-6, of this manual discusses the use of churn drills. If rotary drilling is used, an engineered drilling fluid (mud) designed to prevent caving and minimize intrusion of the drilling fluid into the embankment shall be used. An appendix in ER 1110-1-1807 provides detailed procedures for rotary drilling.

### *Section II* *Drillhole Inspection and Logging*

## 5-7. Objectives

A major part of field investigations is the compilation of accurate borehole logs on which subsequent geologic and geotechnical information and decisions are based. A field drilling log for each borehole can provide an accurate and comprehensive record of the lithology and stratigraphy of soils and rocks encountered in the borehole and other relevant information obtained during drilling, sampling, and in situ testing. To accomplish this objective, an experienced geologist, soils engineer, or civil engineer with good geotechnical training and experience should be present during drilling. The duties of the field inspector include the following:

- a. Making decisions on boring location, depth, and number and quality of samples required.
- b. Observing and describing drilling tools and procedures.
- c. Observing, classifying, and describing geologic materials and their discontinuities.
- d. Selecting and preserving samples.

- e.* Performing field tests on soils (hand penetrometer, torvane).
- f.* Photographing site conditions and rock cores.
- g.* Observing and recording drilling activities and ground water measurements.
- h.* Overseeing and recording instrument installation activities.
- i.* Completing the drilling log, ENG FORM 1836 and/or entering information in BLDM (Nash 1993).
- j.* Recording information and data from in situ tests.

The logs of borings are normally made available to contractors for use in preparing their bids. The descriptions contained on the logs of borings give the contractor an indication of the type of materials to be encountered and their in situ condition. Special care must be taken to ensure a clear differentiation in logs between field observations and laboratory test results. Guidance on soil identification and description, coring, and core logging is provided in the remainder of this section.

## **5-8. Soil Identification and Description**

A thorough and accurate description of soils is important in establishing general engineering properties for design and anticipated behavior during construction. The description must identify the type of soil (clay, sand, etc.), place it within established groupings, and include a general description of the condition of the material (soft, firm, loose, dense, dry, moist, etc.). Characterization of the soils within a site provides guidance for further subsurface exploration, selection of samples for detailed testing, and development of generalized subsurface profiles (Das 1994). Initial field soil classification with subsequent lab tests and other boring data are recorded on the logs of borings. Soils should be described in accordance with ASTM D 2488-93 (ASTM 1996d). For civil works, the most widely used classification is the Unified Soil Classification System (USCS). The USCS outlines field procedures for determining plasticity, dilatancy, dry strength, particle size, and other engineering parameters. The USCS is described by Schroeder (1984) and in Technical Memorandum 3-357 (USAEWES 1982). A number of references provide detailed procedures to evaluate the physical properties of soils, including Cernica (1993), Lambe and Whitman (1969), Terzaghi, Peck, and Mesri (1996), and Means and Parcher (1963). In some cases, a standardized description of color using Munsell charts is useful. Some of the procedures, such as determining dry strength, may be impractical under certain field conditions and may be omitted where necessary. However, the checklists included in the procedure, if followed conscientiously, provide for a thorough description of soils. Examples for presenting soils data on ENG FORM 1836 are shown in Appendix D. Examples of well logs in the Boring Log Data Manager format are also presented in Appendix D.

## **5-9. Coring**

Core drilling, if carefully executed and properly reported, can produce invaluable subsurface information. Basic procedures that should be followed and the information obtained can form the basis for comparison for widely diverse sites and conditions. The following subparagraphs outline procedures to report observations made during coring operations.

*a. Drilling observations.* During the coring operation, a great deal of information is available about the subsurface conditions that may or may not be apparent in the core recovered from the hole. Observation of the drilling action must be made and reported to present as complete a picture as possible of the subsurface conditions.

(1) If coring with water as a circulating medium, the inspector should note the amount of water return relative to the amount being injected through the drill rods and its color. Careful observation of drill water return changes can indicate potential intervals where pressure test takes can be anticipated and correlated. Changes in the color of the return water can indicate stratigraphic changes and degrees of weathering such as clay-filled joints and cavity fillings.

(2) If available, hydraulic pressure being exerted by the drill should be recorded on each run as well as the fluid water pressure. While the drill is turning, the inspector should correlate drilling depths to drilling action (e.g., smooth or rough), increases and decreases applied by the drill operator to the feed control valve, and the rate of penetration. Rod drop depths, which indicate open zones, should be recorded. Changes in drilling rates can be related to changes in composition and/or rock structure and, in areas of poor core recovery, may provide the only indication of the subsurface conditions.

*b. Procedural information.* Regardless of the program undertaken, all logs should at least include the following: size and type of core bit and barrel used; bit changes; size, type, and depth of casing; casing shoe and/or casing bit used; problems or observations made during placement of the casing; change in depth of casing setting during drilling; depth, length, and time for each run; length/depth of pull (the actual interval of core recovered in the core run); amount of core actually recovered; amount of core loss or gain; and amount of core left in the hole (tape check). The inspector should note the presence of a flange on the bottom of a core string because a flange indicates that the core was retrieved from the bottom of the drilled hole. From these data the unaccountable loss, i.e., the core that is missing and unaccounted for, should be computed. Core loss should be shown on the graphic log and by blocks or spacers in the core box at its most likely depth of occurrence based upon the drilling action and close examination of the core. The boring should be cleaned and the total depth taped to determine the amount of cored rock left in the hole on the final run.

## **5-10. Core Logging**

Each feature logged shall be described in such a way that other persons looking at the core log will recognize what the feature is, the depth at which it occurred in the boring, and its thickness or size. They should also be able to obtain some idea of the appearance of the core and an indication of its physical characteristics. The log shall contain all the information obtainable from the core pertaining to the rock as well as discontinuities. Examples for presenting core logging data on ENG FORM 1836 are shown in Appendix D.

*a. Rock description.* Each lithologic unit in the core shall be logged. The classification and description of each unit shall be as complete as possible. A recommended order of descriptions is as follows:

- (1) Unit designation (Miami oolite, Clayton formation, Chattanooga shale).
- (2) Rock type and lithology.
- (3) Hardness, relative strength, or induration..

- (4) Degree of weathering.
- (5) Texture.
- (6) Structure.
- (7) Discontinuities (faults, fractures, joints, seams).
  - (a) Orientation with respect to core axis.
  - (b) Asperity (surface roughness).
  - (c) Nature of infilling or coating, if present.
  - (d) Staining, if present.
  - (e) Tightness.
- (8) Color.
- (9) Solution and void conditions.
- (10) Swelling and slaking properties, if apparent.
- (11) Additional descriptions such as mineralization, inclusions, and fossils.

Criteria for these descriptive elements are contained in Table B-2 (Appendix B). Murphy (1985) provides guidelines for geotechnical descriptions of rock and rock masses. Geological Society Engineering Group Working Party Report (1995) suggests a description and classification scheme of weathered rocks for engineering purposes. Variation from the general description of the unit and features not included in the general description should be indicated at the depth and the interval in the core where the feature exists. These variations and features shall be identified by terms that will adequately describe the feature or variation so as to delineate it from the general description. Features include zones or seams of different color and texture; staining; shale seams, gypsum seams, chert nodules, and calcite masses; mineralized zones; vuggy zones; joints; fractures; open and/or stained bedding planes, roughness, planarity; faults, shear zones, and gouge; cavities, thickness, open or filled, and nature of filling; and core left in the bottom of the hole after the final pull.

*b. Rock quality designation.* A simple and widely used measure of the quality of the rock mass is provided by the Rock Quality Designation (RQD), which incorporates only sound, intact pieces 10 cm (4 in.) or longer in determining core recovery. In practice, the RQD is measured for each core run and reported on ENG Form 1836. Many of the rock mass classification systems in use today are based, in part, on the RQD. Its wide use and ease of measurement make it an important piece of information to be gathered on all core holes. It is also desirable because it is a quantitative measure of core quality at the time of drilling before handling and slaking have had major effect. Deere and Deere (1989) reevaluated the use of RQD from experience gained in the 20 years since its inception. They recommended modifications to the original procedure after evaluating results of field use. Figure 5-1 illustrates the modified procedure of Deere and Deere.

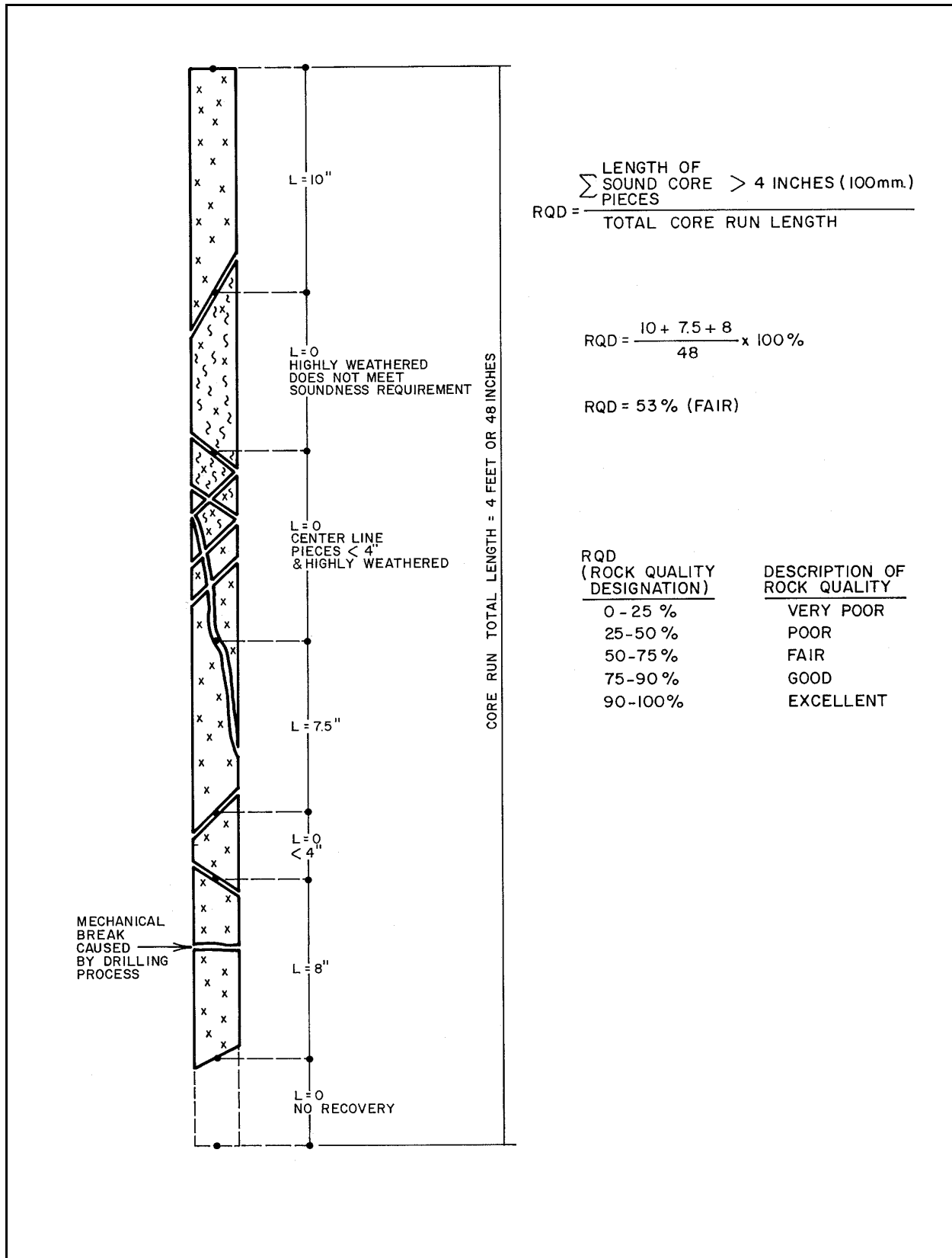


Figure 5-1. Illustration of Deere and Deere (1989) modified procedure for calculating RQD

(1) RQD was originally recommended for NX size (5.474-cm- or 2.155-in.-diam) core, but Deere and Deere expanded its use to the somewhat smaller NQ wireline sizes (4.763 cm or 1-7/8 in.) and to larger wireline sizes up to 8.493 cm (3-11/32 in.) and other core sizes up to 15 cm (6 in.). They discouraged RQD use with the smaller BQ (3.651-cm or 1-7/16-in.) and BX (4.204-cm or 1.655-in.) cores because of core breakage.

(2) Core segment lengths should be measured along the centerline or axis of the core, as illustrated in Figure 5-1.

(3) The inspector should disregard mechanical breaks (breaks caused by drilling action or handling) when calculating RQD.

(4) RQD should be performed at the time the core is retrieved to avoid the effects of postremoval slaking and separation of core along bedding planes, as in some shales.

(5) Emphasis should be placed on core being “sound.” Pieces of core that do not meet the subjective “soundness” test should not be counted. Indicators of “unsound” rock are discolored or bleached grains or crystals, heavy staining, pitting, or weak grain boundaries. Unsound rock is analogous to “highly weathered” rock, which is characterized by weathering extending throughout the rock mass.

Several papers have appeared since Deere and Deere (1989) suggesting alternatives or modified applications of RQD to systems of discontinuities that are perhaps less amenable to analysis by the original procedure. Boadu and Long (1994) established a relation between RQD and fractal dimension (the degree to which a system is self-similar at different scales). The relationships may have application in fracture geometries with complex distributions. Eissa and Sen (1991) suggest alternative analytical methods to RQD when dealing with fracture networks, that is, sets of fractures in more than one direction. Similar alternative approaches to systems of fractures in three dimensions (a volumetric approach) were proposed by Sen and Eissa (1991). Special attention should be paid to the nature of all discontinuities. These are most often what control the engineering behavior of the foundation rock mass and slope stability.

*c. Solution and void conditions.* Solution and void conditions shall be described in detail because these features can affect the strength of the rock and can indicate potential ground water seepage paths. Where cavities are detected by drilling action, the depth to top and bottom of the cavity should be determined by measuring. Filling material, where present and recovered, should be described in detail opposite the cavity location on the log. If no material is recovered from the cavity, the inspector should note the probable conditions of the cavity, as determined by observing the drilling action and the color of the drilling fluid. If drilling action indicates material is present, e.g., a slow rod drop, no loss of drill water, or noticeable change in color of water return, it should be noted on the log that the cavity was probably filled and the materials should be described as well as possible from the cuttings or traces left on the core. If drilling action indicates the cavity was open, i.e., no resistance to the drilling tools and/or loss of drilling fluid, it should be noted on the drilling log. By the same criteria, partially filled cavities should be noted. If possible, filling material should be sampled and preserved. During the field logging of the core at the drilling site, spacers should be placed in the proper position in core boxes to record voids and losses.

*d. Photographic and video record.* A color photographic record of all core samples should be made. Photographs should be taken as soon as possible after retrieving the core samples. The core photographs can be reproduced on 20- by 25-cm (8- by 10-in.) prints, two or three core boxes to a photograph, and the photographic sheets placed in a loose-leaf binder for convenient reference. Photographs often enhance the logged description of cores particularly where rock defects are abundant.

In the event that cores are lost or destroyed, the photographic record becomes the only direct, visual means for review of subsurface conditions without expensive redrilling. A video recording of the drilling operation provides an excellent record of drilling equipment and procedures. Moreover, video may provide a record of critical events or conditions that were not obvious at the time, or occurred too quickly to be recorded manually.

### **5-11. Drilling Log Form and the Boring Log Data Management Program**

All soil and rock drilling logs will be recorded using ENG FORM 1836 as the standard, official log of record. As a general rule, the depth scale on each sheet should normally be 3 m (10 ft) per page and no smaller than 6 m (20 ft). Examples of completed drilling logs are shown in Appendix D. A PC-based, menu-driven boring log data management program (BLDM) is available for free to COE personnel through CEWES-GS-S. The BLDM allows users to create and maintain boring log data, print reports, and create data files which can be exported to a GIS (Nash 1993). Examples of BLDM output are presented in Appendix D.

#### *Section III* *Borehole Examination and Testing*

### **5-12. Borehole Geophysical Testing**

A wide array of downhole geophysical probes is available to measure various formation properties (Tables 4-1 and 4-2). Geophysical probes are not a substitute for core sampling and analysis, however, but they are an economical and valuable supplement to the core sample record. Some very sophisticated analyses of rock mass engineering properties are possible through the use of downhole geophysics. These services are available through commercial logging companies and various Government agencies. Recent developments in microcomputer technology have made it possible to apply procedures known as crosshole tomography to borehole seismic and resistivity data (Cottin et al. 1986; Larkin et al. 1990). Through computer analysis of crosshole seismic and resistivity data, tomography produces a 3-D rendition of the subsurface. The level of detail possible depends upon the distance between holes, the power of the source, and the properties of the rock or soil mass. The method can be used for both indurated and nonindurated geomaterials.

### **5-13. Borehole Viewing and Photography**

The interpretation of subsurface conditions solely by observation, study, and testing of rock samples recovered from core borings often imposes an unnecessary limitation in obtaining the best possible picture of the site subsurface geology. The sidewalls of the borehole from which the core has been extracted offer a unique picture of the subsurface where all structural features of the rock formation are still in their original position. This view of the rock can be important, particularly if portions of rock core have been lost during the drilling operation and if the true dip and strike of the structural features are required. Borehole viewing and photography equipment includes borescopes, photographic cameras, TV cameras, sonic imagery loggers, caliper loggers, and alinement survey devices. Sonic imagery and caliper loggers are discussed in detail in EM 1110-1-1802. Alinement survey services are available from commercial logging or drilling firms and from the U.S. Army Engineer Waterways Experiment Station (CEWES-GG-F). Borehole viewing systems and services are often obtained now from private industry or from the few COE offices that have the capabilities.

#### **5-14. Borehole Camera and Borescope**

Borehole film cameras that have limited focus capability are satisfactory for examining rock features on the sidewalls of the borehole. However, the small viewing area and limited focus reduce the usefulness in borings that have caved or that have cavities. They are best used for examining soft zones for which core may not have been recovered in drilling, for determination of the dip and strike of important structural features of the rock formation, and to evaluate the intrusion of grout into the rock mass. The camera's film must be processed before the images can be examined. The borescope, basically a tubular periscope, has limited use because of its small viewing area, limited depth, and cumbersome operation. It is relatively inexpensive to use, however.

#### **5-15. Borehole TV Camera and Sonic Imagery**

The TV camera has variable focus and is suitable for examining the nature and approximate dimensions of caving sections of open boreholes or boreholes filled with clear water. The TV camera provides both real-time imagery and a permanent record of the viewing session. The sonic imagery (televiewer) system uses acoustic pulses to produce a borehole wall image and can be used in a hole filled with drilling mud. The TV camera is used to examine cavities in the rock such as solution voids in calcareous formations, open cooling joints, and lava tunnels in volcanic rocks, mines, tunnels, and shafts. Most TV systems are capable of both axial (downhole) and radial (sidewall) viewing. The televiewer can be used to distinguish fractures, soft seams, cavities, and other discontinuities. Changes in lithology and porosity may also be distinguished. Specially designed borehole television cameras and sonic imagers or televiewers can be used to determine the strike and dip of discontinuities in the borehole wall. The Corps of Engineers has this capability at the U.S. Army Engineer District, Walla Walla, WES, and the U.S. Army Engineer Division Laboratory, Southwestern.

#### **5-16. Alinement Surveys**

Alinement surveys are often necessary if the plumbness and/or orientation of a hole is important. Older methods employed a compass and photograph system which was relatively easy to use. More modern systems are electronic. Alinement surveys may be critical in deep holes where instrumentation packages are to be installed or where precise determinations of structural features in the rock formation are required.

#### *Section IV* *Exploratory Excavations*

#### **5-17. Test Pits and Trenches**

Test pits and trenches can be constructed quickly and economically by bulldozers, backhoes, pans, draglines, or ditching machines. Depths generally are less than 6 to 9 m (20 to 30 ft), and sides may require shoring if personnel must work in the excavations. Test pits, however, hand dug with pneumatic jackhammers and shored with steel cribbing, can be dug to depths exceeding 18 m (60 ft). Test pits and trenches generally are used only above the ground water level. Test pits that extend below the water table can be kept open with air or electric powered dewatering pumps. Exploratory trench excavations are often used in fault evaluation studies. An extension of a rock fault into much younger overburden materials exposed by trenching is usually considered proof of recent fault activity. Shallow test pits are commonly used for evaluating potential borrow areas, determining the geomorphic history, and assessing cultural resource potential.

## 5-18. Calyx Hole Method

Large-diameter calyx holes have been used successfully on some jobs to provide access for direct observation of critical features in the foundations. These holes are very expensive to drill (possibly \$2,300 per meter or \$700 per foot), so their use is very limited. However, where in situ observation of a very sensitive feature, such as a shear zone or solution feature in the abutment of an arch dam, cannot be achieved reasonably by any other means, the calyx hole may be the procedure of choice.

### *Section V*

#### *Ground Water and Foundation Seepage Studies*

## 5-19. General Investigation

The scope of ground water studies is determined by the size and nature of the proposed project. Efforts can range from broad regional studies at a reservoir project to site-specific studies, such as pumping tests for relief well design, water supply at a recreational area, or pressure tests performed to evaluate the need for foundation grouting. Ground water studies include observations and measurements of flows from springs and of water levels in existing production wells, boreholes, selected observation wells, and piezometers. This information is used with site and regional geologic information to determine water table or piezometric surface elevations and profiles, fluctuations in water table elevations, the possible existence and location of perched water tables, depths to water-bearing horizons, direction and rate of seepage flow, and potential for leakage from a proposed reservoir or beneath an embankment or levee. Complex investigations are made only after a thorough analysis has been made of existing or easily acquired data. Results from ground water and foundation seepage studies provide data needed to design dewatering and seepage control systems at construction projects, indicate the potential for pollution and contamination of existing ground water resources due to project operation, show potential for interference to aquifers by the construction of a project, and determine the chemical and biological quality of ground water and that relationship to project requirements. Investigation and continued monitoring of ground water fluctuations are key dam safety issues.

*a. Wells.* Existing wells located during field geologic reconnaissance should be sounded or water levels obtained from the well owners. Pumping quantities, seasonal variations in ground water and pumping levels, depths of wells and screen elevations, corrosion problems, and any other relevant information should be acquired wherever available. Any settlement records attributable to ground water lowering from pumping should be obtained. This information should be compared with water well records obtained during preliminary studies to develop a complete hydrologic picture for the project area.

*b. Borings.* Water levels recorded on drilling logs are another source of information. However, they may not reflect true water levels, depending on soil types and time of reading after initial drilling. The influence of drilling fluids on water level readings should be kept in mind when evaluating boring data. Loss of drilling fluids can indicate zones of high permeability. Where ground water level information is needed, installation of piezometers or observation wells in borings should be considered.

*c. Piezometers and observation wells.* The most reliable means for determining ground water levels is to install piezometers or observation wells. Piezometers measure excess hydrostatic pressures beneath dams and embankments. All information developed during preliminary studies on the regional ground water regime should be considered in selecting locations for piezometers and observation wells. For types of piezometers, construction details, and sounding devices, refer to EM 1110-2-1908, Part 1,

and TM 5-818-5/AFM 88-5, Chapter 6/NAVFAC P-418. All piezometer borings should be logged carefully and “as built” sketches prepared that show all construction and backfill details (Figure 5-2).

(1) The selection of the screened interval is critical to the information produced, since the water level recorded will be the highest of all intervals within the screen/filter length. Careful evaluation of the conditions encountered in the hole with regard to perched or confined aquifers is essential to a sensible selection of the screened interval and interpretation of the data. One of the greatest benefits of a piezometer or observation well is that it allows for measurement of fluctuations in piezometric levels over time. To take advantage of this benefit, it is necessary to provide for periodic readings. This can be accomplished through manual reading by an automated system, depending on the location and critical importance of the area being monitored.

(2) Other information that can be derived from observation wells and piezometers are temperature and water quality data. Tracer tests can sometimes be conducted to determine the direction and rate of ground water flow.

*d. Springs and surface water.* The water elevation, flow rate, and temperature of all springs located within the project area should be measured. Water should be sampled for chemical analysis to establish a baseline level. Soil or rock strata at the spring should be evaluated to locate permeable horizons. Flow rates at springs should be measured during dry and wet seasons to determine the influence of rainfall on seepage conditions. The elevation of water levels in lakes and ponds should be measured during the wet and dry seasons to evaluate the extent of surface water fluctuations.

*e. Geophysical methods.* Geophysical methods, such as seismic refraction, can be used to determine the depth to saturated material. Depending on the accuracy required and the accuracy of the method, a minimal number of piezometers should be installed to verify the geophysical data. Surface resistivity surveys can indicate the presence of and depth to water (Society of Exploration Geophysicist 1990). Ground penetrating radar can also be used to detect the presence and location of ground water (Annan 1992). Fetter (1988) discusses these and other geophysical methods to characterize the hydrology and hydrogeology of a site.

*f. Tracer testing.* In some areas, especially karst terrains, it is of particular interest to determine flow paths in the ground water system. Although complex, flow paths in karst, where seepage velocities are high, can be evaluated by conducting tracer tests using either environmentally benign dyes or biological tracers such as pollen. The tracer element is introduced into a boring or other access points and monitored at an exit point such as a spring. The travel time from the introduction to detection is recorded. Numerous tests at different locations can be run and a picture of the ground water flow regime developed.

## **5-20. Permeability Testing**

Permeabilities of foundation materials can be determined from slug and pumping tests in piezometers and wells, laboratory tests of undisturbed samples, and pressure tests in rock foundations. The permeability of sands can be roughly estimated from the  $D_{10}$  fraction (TM 5-818-5). Fracture and joint analysis is important in evaluating permeability of rock foundations. General reviews of methods to evaluate permeability of soil and rock in the subsurface include Bentall (1963), Davis and DeWiest (1966), Dawson and Istok (1991), Driscoll (1986), Fetter (1988), Heath (1983), Lohman (1972), and Walton (1970).

<b>DRILLING LOG</b>		DIVISION: ENVIRONMENTAL ENGINEERING		CLIENT: U.S. ARMY - APG D.S.H.E-E.M.D		SHEET 1 OF 2 SHEETS	
1. PROJECT: FTA WESTERN BOUNDARY INVESTIGATION				8. SCREENED INTERVAL: 45.0-50.0 FT		SCREEN TYPE GALVANIZED STEEL 0.010 IN CONTINUOUS SLOT	
2. LOCATION: WB-P1		MSPCS: 656,502 NORTHING 1,541,174 EASTING		9. SAMPLING METHOD (SOIL): 2.0 FT SPLIT SPOON		(AIR) PHOTOVAC MICROTIP HL2000	
3. COUNTY: HARFORD		STATE: MARYLAND		10. DRILLING EQUIPMENT: GUS PECHE BRAT 22R			
4. HOLE NO. (AS SHOWN IN STATE RECORDS) HA-92-0494				11. ELEVATION GROUND WATER (MEASURED FROM GROUND LEVEL) 10.7 FT MSL			
5. NAME OF DRILLER: JAMES MARSH				12. DATE HOLE: STARTED: 14 OCT 92		COMPLETED: 15 OCT 92	
6. DRILLING AGENCY: LAYNE ENVIRONMENTAL SERVICES				13. ELEVATION OF TOP OF HOLE: 40.70 FT		TOP OF RISER 43.41 FT	
7. DEPTH OF HOLE 77.0 FT				NAME OF INSPECTOR MARK A. LEWIS			

DEPTH FEET	CLASSIFICATION OF MATERIALS (DESCRIPTION) USCS	BLOW COUNTS	RECOVERY SAMPLE INTERVAL	SAT. MOIST DAMP	LITHOLOGY	WELL COMPLETION DIAGRAM	REMARKS (Drilling time, water loss, depth of weathering, ect., if significant)	DEPTH FEET
	Light brown sandy silt						Above Ground Completion	
5	Brown silty fine to med. sand with trace fine gravels, subangular SM							5
	White to tan fine sand, poorly sorted SP							
	Tan silty fine sand, subangular SM							
	Tan fine to coarse sands, poorly sorted SP							
10	Brown sandy clay, slightly plastic SC							10
	Brown medium to coarse poorly sorted sand with some fine gravels, subrounded SP							
15								15
20	Light brown silty clay, slight to medium plasticity CL							20
25	Yellowish orange silty fine sand, subangular SM							25
30	Brown sandy clay, medium plasticity SC							30
35	Light brown fine sand, subrounded SP						No Organic Vapor Readings Above Background in Breathing Zone	35
40							Bentonite Slurry Seal	40
45							Filter Pack #2 Sand	45

Figure 5-2. Example of a report-quality log with lithologic, blow count, moisture, and well completion information. Note that the header contains a variety of details concerning this monitoring well (Continued)

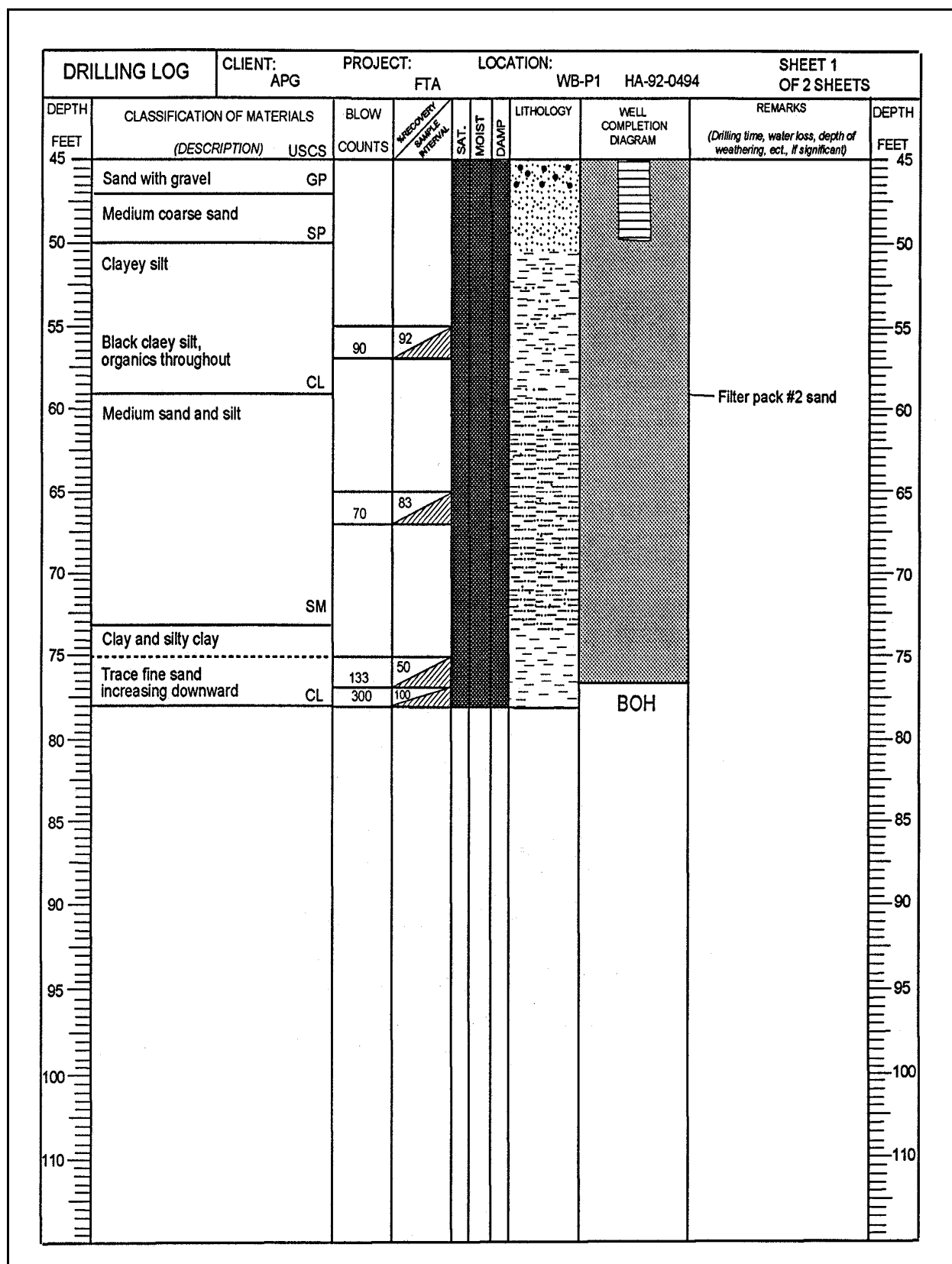


Figure 5-2. (Concluded)

*a. Tests in piezometers or wells.* Permeability tests can easily be made in piezometers or wells. They should be performed as part of piezometer installation procedures, both to obtain permeability information and to assure that the piezometer is working satisfactorily. Appropriate piezometer permeability tests are constant head, falling or rising head, and slug. The information obtained is representative of a smaller volume of material than that tested in pumping tests. However, procedures are simple, costs are low, and results may be useful if interpreted with discretion. Test details are discussed in EM 1110-2-1908 (Part 1), TM 5-818-5, U.S. Department of Interior (1977), Mitchell, Guzikowski, and Villet (1978), and Bennett and Anderson (1982).

*b. Pumping tests.* Pumping tests are the traditional method for determining permeability of sand, gravels, or rock aquifers. Observation wells should be installed to measure the initial and lowered ground water levels at various distances from the pumped well. At known or suspected HTRW sites, disposal of pumped water is a major consideration. For details of pumping tests and analyses, refer to TM 5-818-5. Pumping tests are usually desirable for the following:

- (1) Large or complex projects requiring dewatering.
- (2) Design of underseepage systems for dams or levees.
- (3) Special aquifer studies.
- (4) Projects where water supply will be obtained from wells.
- (5) Projects immediately downstream from existing embankments.

*c. Permeability of rock.* Most rock masses contain, in addition to intergranular pore spaces, complex interconnecting systems of joints, fractures, bedding planes, and fault zones that, collectively, are capable of transmitting ground water. Fracture or joint permeability is normally several magnitudes higher than the matrix permeability of the discrete blocks or masses of rock contained between the joints. The permeability of some rock masses, such as sandstones and conglomerates, is governed by interstitial voids similar to that of soils. Secondary weathering and solutioning of limestones and dolostones may produce large void spaces and exceptionally high permeabilities. Although the permeability of rock results from interconnecting systems of joints, fractures, and formational voids, the equivalent rock mass permeability can frequently be modeled as a uniform porous system. Although it is necessary to keep the hydrologic model manageable, the shortcoming of this approach is that most rock masses are anisotropic with regard to permeability. The influence of this on a practical level is that it is easy to over- or underestimate the ground water effects in rock. As an example, if a pumping test is conducted with monitoring wells oriented along a line perpendicular to the predominant water-bearing joint set, the results will underestimate the radius of influence along the joint set. Therefore, the layout of pumping tests must be well thought out beforehand. At least a preliminary fracture and joint analysis should be conducted prior to laying out a pump test.

*d. Fracture and joint analysis.* Because joint or fracture permeability frequently accounts for most of the flow of water through rocks, an accurate description of in situ fracture conditions of a rock mass is critical to predicting performance of drains, wells, and piezometer responses. Joints typically occur in sets which have similar orientations. There may be three or more sets of joints in a rock mass. Joint sets that occur in the rock mass at the site should be identified and the preferred orientation and range in orientation of each joint set recorded. Features such as joint orientation, spacing, joint width, and the degree and type of secondary mineral filling should be carefully noted for each joint set. Once all joint

sets of a site have been identified and evaluated, their relative importance to ground water flow should be assessed. Joints and fractures can be evaluated by developing the structure and stratigraphy of the site from accessible outcrops and from borehole logs.

## 5-21. Pressure Tests

*a.* Pressure tests are performed to measure the permeability of zones within rock masses. Pressure test results are used in assessing leakage in the foundation and as a guide in estimating grouting requirements. Pressure tests are typically conducted during exploratory core drilling and are a relatively inexpensive method of obtaining important hydrogeologic information about a rock mass. Hydraulic pressure testing should be considered an integral part of the exploratory core drilling process in all cases where rock seepage characteristics could affect project safety, feasibility, or economy. The testing interval is typically 1.5 to 3 m (5 to 10 ft) but may be varied to fit specific geological conditions observed during the core drilling operations. Zones to be tested should be determined by (1) examining freshly extracted cores, (2) noting depths where drilling water was lost or gained, (3) noting drill rod drop, (4) performing borehole or TV camera surveys, and (5) conducting downhole geophysical surveys. In rock with vertical or high angle joints, inclined borings are necessary to obtain meaningful results. Types of tests and test procedures are described in Ziegler (1976), U.S. Department of Interior (1977), and Bertram (1979).

*b.* Pressures applied to the test section during tests should normally be limited to 23 kilo Pascals (kP) per meter (1 psi per foot) of depth above and 13 kP/meter (0.57 psi/foot) of depth below the piezometric surface. The limit was established to avoid jacking and damage to rock formations. The limit is conservative for massive igneous and metamorphic rocks. However, it should be closely adhered to for tests in horizontally bedded sedimentary and other similar types of formations. Naturally occurring excess water pressures (artesian) should be taken into account in computations for limiting test pressures. Where the test intervals are large, a reduction in total pressure may be necessary to prevent jacking of the formation within the upper portion of the test section.

*c.* An important, but often unrecognized, phenomenon in pressure testing is joint dilation and contraction as pressure is applied and released. In the case of a dam project, it is desirable to use pressures that will correspond to future reservoir conditions. Joint dilation can frequently be observed by conducting a "holding" test. The fall in pressure is observed and a plot of pressure versus time is made. The pressure should quickly drop to near the surrounding piezometric level if the joint openings remain the same width. The common observation of a slow pressure decay in pressure holding tests indicates joint closure with reduction in pressure.

*d.* Qualitative evaluations of leakage and grout requirements can be made from raw pressure test data (Ziegler 1976, U.S. Department of Interior 1977, Bertram 1979). Most analyses of this type assume laminar flow rather than turbulent flow. This assumption can be verified by conducting pressure tests on the same interval at several different pressures. If the water take is directly proportional to the total applied pressure, laminar flow can be assumed. If pressure test data are converted into values of equivalent permeability or transmissivity, calculations can be performed to estimate seepage quantities. Wherever possible, such results should be compared with data from completed projects where similar geologic conditions exist.

*Section VI*  
*In situ Testing to Determine Geotechnical Properties*

## 5-22. In Situ Testing

In situ tests are often the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Table 5-2 lists in situ tests and their purposes. In situ rock tests are performed to determine in situ stresses and deformation properties (moduli) of the jointed rock mass, shear strength of jointed rock masses or critically weak seams within the rock mass, and residual stresses along discontinuities or weak seams in the rock mass. Pressure tests have been discussed in Section V (paragraph 5-20) of this manual.

**Table 5-2**  
**In Situ Tests for Rock and Soil**

Purpose of Test	Type of Test	Applicability to	
		Soil	Rock
Shear strength	Standard penetration test (SPT)	X	
	Field vane shear	X	
	Cone penetrometer test (CPT)	X	
	Direct shear	X	
	Plate bearing or jacking	X	X <sup>1</sup>
	Borehole direct shear <sup>2</sup>	X	
	Pressuremeter <sup>2</sup>		X
	Uniaxial compressive <sup>2</sup>		X
Bearing capacity	Borehole jacking <sup>2</sup>		X
Bearing capacity	Plate bearing	X	X <sup>1</sup>
	Standard penetration	X	
Stress conditions	Hydraulic fracturing	X	X
	Pressuremeter	X	X <sup>1</sup>
	Overcoring		X
	Flatjack		X
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking <sup>2</sup>		X
	Chamber (gallery) pressure <sup>2</sup>		X
Mass deformability	Geophysical (refraction)	X	X
	Pressuremeter or dilatometer	X	X <sup>1</sup>
	Plate bearing	X	X
	Standard penetration	X	
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking <sup>2</sup>		X
	Chamber (gallery) pressure <sup>2</sup>		X
Relative density	Standard penetration	X	
	In situ sampling	X	
	Cone <sup>2</sup> penetrometer	X	
Liquefaction susceptibility	Standard penetration	X	
	Cone penetrometer test (CPT) <sup>2</sup>	X	

<sup>1</sup> Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.

<sup>2</sup> Less frequently used.

*a. Soils, clay shales, and moisture-sensitive rocks.* Interpretation of in situ tests in soils, clay shales, and moisture-sensitive rocks requires consideration of the drainage that may occur during the test. Consolidation during testing makes it difficult to determine whether the test results correspond to unconsolidated-undrained, consolidated-undrained, consolidated-drained conditions, or intermediate conditions between these limiting states. The cone penetrometer test is very useful for detecting soft or weak layers and in quantifying undrained strength trends with depth. Interpretation of in situ test results

requires complete evaluation of the test conditions and the limitations of the test procedure. ASTM D 3877-80 (ASTM 1996h) is the standard laboratory method for evaluating shrink/swell of soils due to subtraction or addition of water.

*b. Rock.* Rock formations are generally separated by natural joints, bedding planes, and other discontinuities resulting in a system of irregularly shaped blocks that respond as a discontinuum to various loading conditions. Response of a jointed rock mass to imposed loads involves a complex interaction of compression, sliding, wedging, rotation, and possibly fracturing of individual rock blocks. Individual blocks generally have relatively high strengths, whereas the strength along discontinuities is normally reduced and highly anisotropic. Commonly, little or no tensile strength exists across discontinuities. As a result, resolution of forces within the system generally cannot be accomplished by ordinary analytical methods. Large-scale, in situ tests tend to average out the effect of complex interactions. In situ tests in rock are generally expensive and should be reserved for projects with large, concentrated loads. Well-conducted tests, however, may be useful in reducing overly conservative, costly assumptions. Such tests should be located in the same general area as a proposed structure and test loading should be applied in the same direction as the proposed structural loading.

### 5-23. In situ Tests to Determine Shear Strength

Table 5-3 lists in situ tests that are useful for determining the shear strength of subsurface materials. In situ shear tests are discussed and compared by Nicholson (1983b) and Bowles (1996).

**Table 5-3**  
**In Situ Tests to Determine Shear Strength**

Test	For		Reference	Remarks
	Soils	Rocks		
Standard penetration	X		EM 1110-2-1906 Appendix C	Use as index test only for strength. Develop local correlations. Unconfined compressive strength in tons/square foot is often 1/6 to 1/8 of N-value
Direct shear	X	X	RTH 321 <sup>1</sup>	Expensive; use when representative undisturbed samples cannot be obtained
Field vane shear	X		EM 1110-2-1906, Appendix D, Al-Khafaji and Andersland (1992)	Use strength reduction factor
Plate bearing	X	X	ASTM <sup>2</sup> Designation D 1194 ASTM SPT 479 <sup>3</sup>	Evaluate consolidation effects that may occur during test
Uniaxial compression		X	RTH 324 <sup>1</sup>	Primarily for weak rock; expensive since several sizes of specimens must be tested
Cone penetrometer test (CPT)	X		Schmertmann (1978a); Jamiolkowski et al. (1982)	Consolidated undrained strength of clays; requires estimate of bearing factor, $N_c$

<sup>1</sup> Rock Testing Handbook (USAEWES 1993).

<sup>2</sup> American Society for Testing and Materials (ASTM 1996a).

<sup>3</sup> Special Technical Publication 479 (ASTM 1970).

a. *The Standard Penetration Test (SPT).* The SPT is useful for preliminary appraisals of a site (Bowles 1996). The N-value has been empirically correlated with liquefaction susceptibility under seismic loadings (Seed 1979). The N-value is also useful for pile design. In cohesive soils, the N-value can be used to determine where undisturbed samples should be obtained. The N-value can also be used to estimate the bearing capacity (Meyerhof 1956; Parry 1977), the unconfined compressive strength of soils (Mitchell, Guzikowski, and Villet 1978), and settlement of footings in soil (Terzaghi, Peck, and Mesri 1996).

b. *The Becker Penetration Test.* The Becker drill (paragraph 5-5(2)) provides estimates of in situ soil strength and other properties similarly to the SPT, including coarse grained soils like gravel. The Becker penetration test was described in Harder and Seed (1986). The test consists of counting the number of hammer blows required to drive the casing 1 ft into the soil, for each foot of penetration. The test uses both open casing and plugged bits, commonly with a 14-cm (5.5-in.) or 17-cm (6.6-in.) OD casing and bits. Correlations of Becker blowcounts with SPT blowcounts have been developed to allow the use of Becker data in foundation investigations and in evaluation of liquefaction potential in coarse-grained soils under seismic loading.

c. *Direct shear tests.* In situ direct shear tests are expensive and are performed only where doubt exists about available shear strength data and where thin, soft, continuous layers exist within strong adjacent materials. The strength of most rock masses, and hence the stability of structures, is often controlled by the discontinuities separating two portions of the rock mass. Factors controlling the shear of a discontinuity include the loads imposed on the interface, the roughness of the discontinuity surfaces, the nature of the material between the rock blocks, and the pore water pressure within the discontinuity (Nicholson 1983b). In situ direct shear tests measure the shear strength along a discontinuity surface by isolating a block of rock and the discontinuity, subjecting the specimen to a normal load perpendicular to and another load (the shear load) parallel to the plane. The advantages of the direct shear test are: (1) its adaptability to field conditions, i.e., a trench, an adit, a tunnel, or in a calyx hole; (2) it is ideal for determining discontinuity shear strength because the failure plane and direction of failure are chosen before testing, accommodating anisotropic conditions; and (3) it allows for volume increases along the failure plane. Disadvantages of direct shear tests are their expense, the fact that they measure strength along only one potential failure plane, and the sometimes nonuniform application of normal stress during shearing. For the latter reasons, some engineers favor the triaxial compression test, which also can be performed in situ, for determination of shear strength (Ziegler 1972). The direct shear test measures peak and residual strength as a function of stress normal to the shear plane. Results are usually employed in limiting equilibrium analysis of slope stability problems or for stability analysis of foundations for large structures such as dams. Where field evidence suggests that only residual strengths can be relied on, either in a thin layer or in a mass, because of jointing, slickensiding, or old shear surfaces, in situ direct shear tests may be necessary. Few in situ direct shear tests are performed on soils, but they may be justified on clay shales, indurated clays, very soft rock, and on thin, continuous, weak seams that are difficult to sample. Ziegler (1972) and Nicholson (1983a,b) discuss the principles and methods of performing in situ direct shear tests. The Rock Testing Handbook (RTH) method RTH 321-80 (USAEWES 1993) provides the suggested method for determining in situ shear strength using the direct shear apparatus.

d. *Field vane shear tests.* Field vane tests performed in boreholes can be useful in soft, sensitive clays that are difficult to sample. The vane is attached to a rod and pushed into the soft soil at the bottom of the borehole. The assembly is rotated at a constant rate and the torque measured to provide the unconsolidated, undrained shear strength. The vane can be reactivated to measure the ultimate or residual strength (Hunt 1984). The vane shear test results are affected by soil anisotropy and by the

presence of laminae of silt or sand (Terzaghi, Peck, and Mesri 1996). Shear is applied directionally. Failure of the soil occurs by shearing of horizontal and vertical surfaces. See Al-Khafaji and Andersland (1992) for a discussion of effects of soil anisotropy. The test may give results that are too high. Factors to correct the results are discussed in Bjerrum (1972) and Mitchell, Guzikewski, and Villet (1978). The test has been standardized as ASTM method D 2573 (ASTM 1996e).

*e. Plate bearing tests.* Plate bearing (plate-load) tests can be made on soil or soft rock. They are used to determine subgrade moduli and occasionally to determine strength. The usual procedure is to jack-load a 30- or 76-cm (12- or 30-in.) diam plate against a reaction to twice the design load and measure the deflection under each loading increment. The subgrade modulus is defined as the ratio of unit pressure to unit deflection, or force/length<sup>3</sup> (Hunt 1984). Because of their cost, such tests are normally performed during advanced design studies or during construction.

*f. Cone penetrometer test or dutch cone.* The Cone Penetrometer Test (CPT) can provide detailed information on soil stratigraphy and preliminary estimations of geotechnical properties. Based on the soil type as determined by the CPT (Douglas and Olsen 1981) or adjacent boring, the undrained strength can be estimated for clays (Jamiolkowski et al. 1982, Schmertmann 1970), and the relative density (and friction angle) estimated for sands (Durgunoglu and Mitchell 1975; Mitchell, Guzikewski, and Villet 1978; Schmertmann 1978b). For clays, a bearing factor,  $N_c$ , must be estimated to calculate the undrained strength from the CPT cone resistance and should be close or slightly greater than the CPT sleeve friction resistance if the soil is not sensitive or remolded (Douglas and Olsen 1981). The calculated undrained strength as well as the change of undrained strength with depth can both be used with several techniques to estimate the overconsolidation ratio (OCR) (Schmertmann 1978a). For sands, the relative density can be estimated if the overconsolidation conditions (i.e., lateral stress ratio) and vertical effective stress are known. The friction angle can also be estimated but also depends on the cone surface roughness and the assumed failure surface shape (Durgunoglu and Mitchell 1975). The mechanical (i.e., Dutch) cone is performed at a depth interval of 20 cm (8 in.) using hydraulic gages which measure the force from an inner rod directly in contact with the end of the probe. The electric (PQS or VGRO) cone is pushed at a constant speed for 1-m intervals while electronically measuring cone and friction resistance continuously.

## 5-24. Tests to Determine In Situ Stress

Table 5-4 lists the field tests that can be used to determine in situ stress conditions. The results are used in finite element analyses, estimating loading on tunnels, determining rock burst susceptibility in excavations, and identifying regional active and residual stresses. Stresses occur as a result of gravity forces, actively applied geologic forces such as regional tectonics, and from stored residual-strain energy. Stress is measured to determine the effect on foundations of changes in loading brought about by excavation or construction. Where a confining material has been removed by natural means or by excavation, the remaining material tends to approach a residual state of stress. In a majority of projects, the major principal stress is vertical, i.e., the weight of the overlying material. However, it has been found from measurements made throughout the world that horizontal stresses in the near-surface vicinity, defined as 30 m (100 ft) or less, can be one and one-half to three times higher than the vertical stress. Recognition of this condition during the design phase of investigations is very important. Where high horizontal stresses occur at a project site, the stability of cut slopes and tunnel excavations is affected. In situ testing is the most reliable method for obtaining the magnitude and direction of stresses. The three most common methods for determining in situ stresses are the overcoring, hydrofracture, and flatjack techniques.

**Table 5-4**  
**In Situ Tests to Determine Stress Conditions**

Test	Soils	Rocks	Bibliographic Reference	Remarks
Hydraulic fracturing	X		Leach (1977) Mitchell, Guzikowski, and Villet (1978)	Only for normally consolidated or slightly consolidated soils
Hydraulic fracturing		X	RTH 344 <sup>1</sup> Goodman (1981) Hamison (1978)	Stress measurements in deep holes for tunnels
Vane shear	X		Blight (1974)	Only for recently compacted clays, silts
Overcoring techniques		X	RTH 341 <sup>1</sup> Goodman (1981) Rocha (1970)	Usually limited to shallow depth in rock
Flatjacks		X	RTH 343 <sup>1</sup> Deklotz and Boisen (1970) Goodman (1981)	
Uniaxial (tunnel) jacking	X	X	RTH 365 <sup>1</sup>	May be useful for measuring lateral stresses in clay shales and rocks, also in soils
Pressuremeter (Menard)	X		Al-Khafaji and Andersland (1992), Hunt (1984)	

<sup>1</sup> Rock Testing Handbook (USAEWES 1993).

*a. Overcoring method.* Possibly the most common method used for measuring in situ stresses in rock is overcoring, a stress-relief technique. An NW (75.7 mm (2.980 in.)) core hole is drilled, instrumented, and redrilled with a larger core barrel. The overcoring decouples the rock surrounding the instrument package from the natural stress field of the in-place formation. The change in strain recorded by the instruments is then converted to stress by using the elastic modulus of the rock determined from laboratory tests. At least three separate tests must be made in the rock mass in nonparallel boreholes. A detailed description of the field test is provided by RTH 341-80 (USAEWES 1993). The overcoring method is hampered by the necessity for many instrument lead wires that may be broken during testing. The practical maximum depth of testing is usually less than 45 m (150 ft).

*b. Flatjack method.* In the flatjack method, a slot is bored or cut into the rock wall midway between two inscribed points. Stresses present in the rock will tend to partially close the slot. A hydraulic flatjack is then inserted and grouted into the slot, and the rock is jacked back to its original position as determined by the inscribed points. The unit pressure required is a measure of the in situ stress. Flatjacks installed at different orientations provide a measure of anisotropy (Hunt 1984). The value recorded must be corrected for the influence of the tunnel excavation itself. Flatjack tests require an excavation or tunnel. The high cost for constructing the opening usually precludes this technique as an indexing tool except where the size of the structure and complexity of the site dictate its use.

*c. Hydrofracture method.* The hydrofracture method has been used in soils and rock. A section of hole is isolated with packers at depth, and an increasingly higher water pressure is applied to the zone. A point will be reached where the pressure begins to level off, and there is a marked increase in water take.

This indicates that a crack in the formation has opened, and the threshold pressure has been reached. The threshold pressure measures the minor principal stress component. The orientation is then obtained by an impression packer. This procedure then gives the intensity and direction of the minor principal stress, which is perpendicular to the crack. The hydrofracture method has no particular depth limitation, but drilling deep holes can be very expensive. This expense can often be circumvented by using holes that have been drilled for other purposes. Evidence indicates that stresses measured within 30 m (100 ft) or more of ground surface may not always reflect the actual stress magnitude or orientation at depth. This may be true particularly in areas where closely jointed and weathered surface rock formations are decoupled from the deeper, more intact rock.

## 5-25. Tests to Determine In Situ Deformation

Deformation characteristics of subsurface materials are of major importance in dynamic and seismic analyses for dams and other large structures, static design of concrete gravity and arch dams, tunnels, and certain military projects. Geotechnical investigations for such purposes should be planned jointly by geotechnical personnel and structural engineers. Deformation properties are normally expressed in terms of three interdependent parameters: Young's modulus, shear modulus, and Poisson's ratio. These parameters assume that materials are linear, elastic, homogeneous, and isotropic. In spite of this limitation, these parameters are often used to describe the deformation properties of soil and rock. Large-scale tests (e.g., tunnel jacking) are frequently used because they reduce the effect of nonhomogeneity. Multiple tests, with different orientations, can be used to determine the anisotropy of the deformation properties. Soils, in particular, tend to be nonlinear and inelastic. As a result, their properties are often strain dependent, i.e., moduli determined at low strain levels can be substantially different from those determined at high strain levels. The fact that sample disturbance, particularly in soils, can substantially affect the deformation properties serves as the primary reason for using in situ tests in soils. Table 5-5 lists the in situ tests used to determine one or more of the deformation parameters. Some test results are difficult to relate to the fundamental parameters but are used directly in empirical relationships (Table 5-6). Deformation properties of a jointed rock mass are very important if highly concentrated loadings are directed into the abutments of arch dams in directions that are tangent to the arches at the abutments. In these cases, the ratio of the deformation modulus of the abutment rock to that of the concrete in the dam must not be so low as to cause adverse tensile stresses to develop within the concrete dam. One problem often encountered in conducting in situ deformation tests is the need to include representative sizes of the jointed rock mass in the test, particularly if the joint spacing is moderately large (e.g., 0.6 to 0.9 m or 2 to 3 ft). This problem has been solved in some instances by excavating a chamber in rock, lining it with an impermeable membrane, and subjecting it to hydraulic pressure to load the rock over relatively large areas.

*a. Chamber tests.* Chamber tests are performed in large underground openings, such as exploratory tunnels. Preexisting openings, such as caves or mine chambers, can be used if available and applicable to project conditions. The opening is lined with an impermeable membrane and subjected to hydraulic pressure. Instrumented diametrical gages are used to record increases in tunnel diameter as the pressure load increases. The test is performed through several load-unload cycles. The data are subsequently analyzed to develop load-deformation curves from which a deformation modulus can be selected. The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings. The chamber test method is described by RTH 361-89 (USAEWES 1993).

**Table 5-5**  
**In Situ Tests to Determine Deformation Characteristics**

Test	For		Reference	Remarks
	Soils	Rocks		
Geophysical refraction, cross-hole and downhole	X	X	EM 1110-1-1802	For determining dynamic Young's Modulus, E, at the small strain induced by test procedure. Test values for E must be reduced to values corresponding to strain levels induced by structure or seismic loads
Pressuremeter	X	X	RTH 362 <sup>1</sup> Baguelin, Jezequel, and Shields (1978) Mitchell, Guzikowski, and Villet (1978)	Consider test as possibly useful but not fully evaluated. For soils and soft rocks, shales, etc.
Chamber test	X	X	Hall, Newmark, and Hendron (1974) Stagg and Zienkiewicz (1968)	
Uniaxial (tunnel) jacking	X	X	RTH 365 <sup>1</sup> Stagg and Zienkiewicz (1968)	
Flatjacking		X	RTH 343 <sup>1</sup> Deklotz and Boisen (1970) Goodman (1981)	
Borehole jack or dilatometer		X	RTH 363 <sup>1</sup> Stagg and Zienkiewicz (1968)	
Plate bearing		X	RTH 364 <sup>1</sup> ASTM STP 479 <sup>2</sup> Stagg and Zienkiewicz (1968)	
Plate bearing	X		MIL-STD 621A, Method 104	
Standard penetration	X		Hall, Newmark, and Hendron (1974)	Correlation with static or effective shear modulus, in Mpa (psi), of sands; settlement of footings on clay. Static shear modulus of sand is approximately: $G_{eff} = 1960 N^{0.51}$ in Mpa (psi); N is SPT value

<sup>1</sup> Rock Testing Handbook (USAEWES 1993).

<sup>2</sup> American Society for Testing and Materials, Special Technical Publication 479 (ASTM 1970).

**Table 5-6**  
**Correlations Between Field Tests for Soils, Material Characteristics, and Structural Behavior**

Field Test	Correlation With	Remarks
1 x 1-ft plate load test	Modulus of subgrade reaction. Settlement of footings on sand	Mitchell, Guzikowski, and Villet (1978)
Load test for radar towers	Young's modulus of subgrade soils	MIL-STD-621A
Standard penetration	Settlement of footings and mats on sand; shear modulus	TM 5-818-1; Hall, Newmark, and Hendron (1974) Meyerhof (1956) Parry (1977) U.S. Army Engineer Waterways Experience Station (1954)
N-value		
Cone penetrometer test	$\bar{\sigma}$ of sands; settlement of footings on sand; relative density	Mitchell, Guzikowski, and Villet (1978) Mitchell and Lunne (1978) Schmertmann (1978b) Durgunoglu and Mitchell (1975) Schmertmann (1970) Schmertmann (1978a,b)

*b. Uniaxial jacking test.* An alternative to chamber tests is the uniaxial jacking test (RTH 365-80 (USAEWES 1993)). The test uses a set of diametrically opposed jacks to test large zones of soil and rock. This method produces nearly comparable results with chamber tests without incurring the much greater expense. The test determines how foundation materials will react to controlled loading and unloading cycles and provides data on deformation moduli, creep, and rebound. The uniaxial jacking test is the preferred method for determining deformation properties of rock masses for large projects.

*c. Other deformation tests.* Other methods for measuring deformation properties of in situ rock are anchored cable pull tests, flatjack tests, borehole jacking tests, and radial jacking tests. The anchored cable pull test uses cables, anchored at depth in boreholes, to provide a reaction to large slabs or beams on the surface of the rock. The test is expensive and difficult to define mathematically but offers the advantages of reduced shearing strains and larger volumes of rock being incorporated in the test. Flatjack tests are flexible, and numerous configurations may be adopted. In relation to other deformation tests, the flatjack test is relatively inexpensive and useful where direct access is available to the rock face. Limitations to the method involve the relatively small volume of rock tested and the difficulty in defining a model for calculation of deformation or failure parameters.

(1) The borehole jack ("Goodman" jack), or dilatometer, and the Menard pressuremeter (Terzaghi, Peck, and Mesri 1996; Al-Khafaji and Andersland 1992; and Hunt 1984), which are applied through a borehole, have the primary advantage that direct access to the rock or soil face is not required. The dilatometer determines the deformability of a rock mass by subjecting a section of a borehole to mechanical jack pressure and measuring the resultant wall displacements. Elastic and deformation moduli are calculated. The pressuremeter performs a similar operation in soils and soft rock. The development of a mathematical model for the methods has proved to be more difficult than with most deformation measurement techniques.

(2) Radial jacking tests (RTH 367-89 (USAEWES 1993)) are similar in principle to the borehole jacking tests except that larger volumes of rock are involved in the testing. Typically, steel rings are placed within a tunnel with flatjacks placed between the rings and the tunnel surfaces. The tunnel is loaded radially and deformations are measured. The method is expensive but useful and is in the same category as chamber tests. All methods of deformation measurements have inherent advantages and disadvantages, and thus selection of test methods must be dictated by the nature of the soil or rock mass, the purpose of the test, and the magnitude of the project. Care must be exercised and limitations recognized in the interpretation and use of measurements of deformation.

## **5-26. Determination of Dynamic Moduli by Seismic Methods**

Seismic methods, both downhole and surface, are used on occasion to determine in-place moduli of soil and rock (see Table 5-2). The compressional wave velocity is mathematically combined with the mass density to estimate a dynamic Young's modulus, and the shear wave velocity is similarly used to estimate the dynamic rigidity modulus. However, because particle displacement is so small and loading is transitory during these seismic tests, the resulting modulus values tend to be too high. The seismic method of measuring modulus should not be used in cases where a reliable static modulus value can be obtained. Even where the dynamic modulus is to be used for earthquake analyses, the modulus derived from seismic methods is too high. The moduli and damping characteristics of rock are strain dependent, and the strains imposed on the rock during seismic testing are several orders of magnitude lower than those imposed by a significant earthquake. Generally, as the strain levels increase, the shear modulus and Young's modulus decrease and the damping increases. Consideration of these factors is necessary for earthquake analyses.

*Section VII*  
*Backfilling of Holes and Disposition of Samples and Cores*

### **5-27. Backfilling Boreholes and Exploratory Excavations**

Except where the hole is being preserved for future use, all boreholes and exploratory excavations should be backfilled. The reasons for backfilling holes are to: eliminate safety hazards for personnel and animals, prevent contamination of aquifers, minimize underseepage problems of dams and levees, and minimize adverse environmental impacts. Many states have requirements regarding backfilling boreholes; therefore, appropriate state officials should be consulted. Holes preserved for the installation of instrumentation, borehole examination, or downhole geophysical work should be backfilled when no longer needed. As a minimum, borings that are preserved for future use should be protected with a short section of surface casing, capped, and identified. Test pits, trenches, and shafts should be provided with suitable covers or barricades until they are backfilled. Where conditions permit, exploratory tunnels may be sealed in lieu of backfilling. Procedures for backfilling boreholes and exploratory excavations are discussed in Appendix F, Chapter 10, of this manual.

### **5-28. Disposition of Soil Samples**

Soil samples may be discarded once the testing program for which they were taken is complete. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of*. Soil samples are not normally retained for long periods, because even the most careful sealing and storing procedures cannot prevent the physical and chemical changes that, in time, would invalidate any subsequent test results. Requirements for the disposition of soil samples from plant pest quarantined areas are specified in ER 1110-1-5.

### **5-29. Disposition of Rock Cores**

All exploratory and other cores not used for test purposes shall be properly preserved, boxed, and stored in a protected storage facility until disposal. The following procedures govern the ultimate disposition of the cores.

*a. Care and storage.* Filled core boxes can be temporarily protected at the drilling site by wrapping them in plastic sheeting and preventing direct contact of the boxes with the ground. Exploratory or other cores, regardless of age, will be retained until the detailed logs, photographs, and test data have been made a matter of permanent record. Precautions shall be taken to ensure against the disposal, destruction, or loss of cores that may have a bearing on any unsettled claim. Such cores shall be retained until final settlement of all obligations and claims. They then will be disposed of in accordance with the procedures outlined in the following text.

*b. Disposal.* Cores over 15 cm (6 in.) in diameter may be discarded after they have served their special purpose. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of*. In a case where the project is deauthorized, all associated cores may be discarded. When a project has been completed and final settlement has been made with the contractors and others concerned, all cores, except those related to future construction, and a few selected cores representative of foundation and abutment conditions, may be discarded. Selected cores, retained after the completion of a project, and additions thereto, may be discarded or otherwise disposed of 5 years after final completion of the project, provided no unforeseen foundation or abutment conditions have developed. After cores are disposed of, core boxes should be salvaged for reuse if their condition permits.